The Use of Seismic Fuses in Design and Construction for Seismic Zones

1 Introduction

The Mediterranean region is prone to earthquakes. Events in Algeria and Turkey have made the news recently, but these are only symptoms of an endemic problem, which continually threatens life and property on a large and destructive scale. Enough is known about the science to prevent much of the devastation, and building codes have been developed to minimise, though not eliminate the risks.

Expensive methods of construction have been adopted in parts of the USA and Japan, but these may not provide realistic solutions in areas such as the Mediterranean region. Concrete and brick are used extensively because of cost and availability, but these materials are often used without reference to seismic design principles.

Eurocode 8 is a set of design guidelines for the construction industry, which helps the fundamental design principles to be applied in seismic zones. One section in particular gives limits for certain properties of reinforcement steel. Unfortunately, steel commonly used in construction has been shown to fall outside these limits, thus making it difficult for seismic design codes of practice to be adhered to.

One of these key factors is “over-strength”, i.e. the degree to which the actual strength of reinforcement steel exceeds its nominal value. This is not a problem in normal construction (the stronger the better), but in designing for seismic conditions, the way in which certain parts of a structure yield and dissipate energy is important in minimising damage and in preventing catastrophic collapse.

“Seismic Fuses” are steel components with known tensile properties, which are produced under rigorous control criteria and are fitted between sections of reinforcement bar at dissipative locations in the structure. By fitting these where the need for control arises, they ensure that the failure mechanism devised would be attained. They also allow the use of lower over-strength factors in the design of non-dissipative zones, where common reinforcement steel should be used.

Thus, “Seismic Fuses”, strategically distributed throughout the structure, provide design engineers with a reliable tool for earthquake-resistant design and code verification. In this way Eurocode 8 requirements for the design of highly ductile structures can be more easily met. Moreover, high quality steel is needed only in smaller quantities, thus the solution is not expensive. Consequently, the detailing of earthquake resistant structures results in more economical designs, whilst the level of confidence in their dynamic response is increased.

2 State of the Art

2.1 Different Approaches

Several approaches have been developed to seismic construction over the last few years, and a large number of products have been promoted. These fall into various categories, viz:

Yielding devices

- Seam-welded plate cantilever
- Constricted-tube device
- Bulged shaft device

Bearings

- High damping rubber bearings
- Natural rubber bearings
- Lead-rubber bearings, e.g DIS Seismic (Base) Isolator
- Neoprene bearings

Dampers

- Visco-elastic dampers
- Shape memory alloy dampers
- Viscous dampers
  - Diagonal brace dampers
  - Base isolation dampers
- Friction dampers

Friction systems

- Friction pendulum system
- General slider

Other

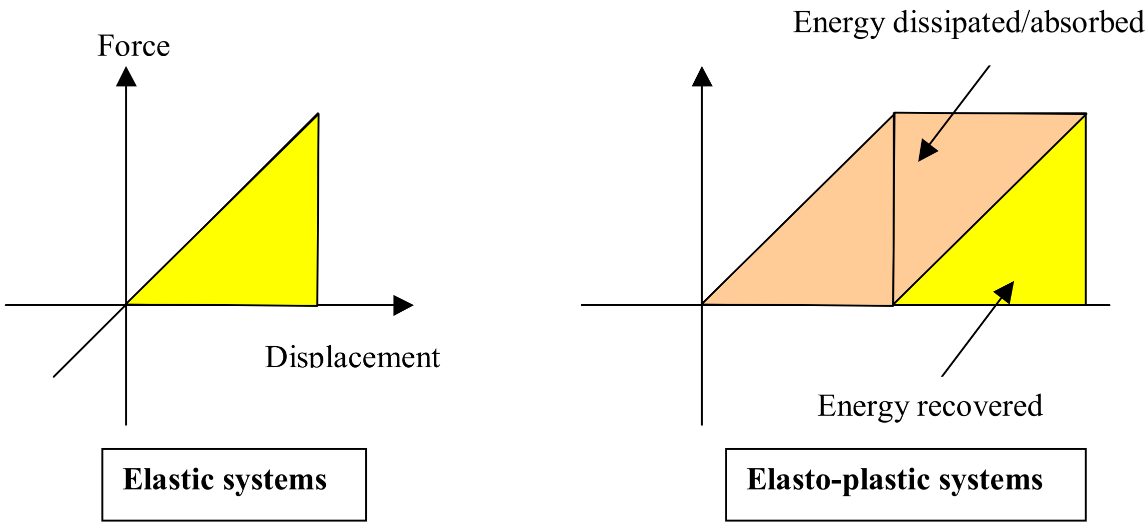
- Quake-wrap external binding

2.2 Reinforced Concrete

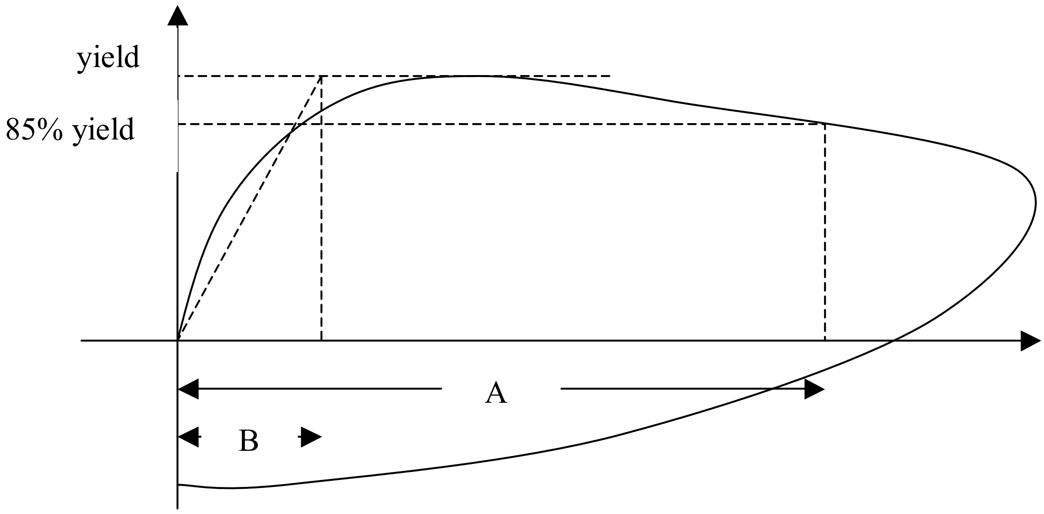
Due to the nature of earthquakes source mechanisms, there is considerable uncertainty regarding the level of displacement demand which will be imposed on structures. Such demand can range from moderate to extremely high, so that using the same (linear elastic) design methodology as for static loads (gravity and wind) would result in very "heavy" designs. These would be neither economical nor practical. Moreover, large accelerations could result from the elastic response, which could endanger lives and cause excessive non-structural damage.

Earthquake designs usually respond in a non-linear fashion. A much lower seismic load is accommodated in the elastic response, but the capacity is provided for the structure to deform inelastically, without significant loss of strength. This results in more economic designs, and also safeguards the structure from higher than predicted seismic forces. This inelastic deformation is achieved through the development of a "plastic hinge" where the energy is dissipated through hysteresis. Thus the deformation competence of structural members (e.g. beams) is as important as their strength.

The capacity of structures or sections thereof to deform without significant loss of strength, known as ductility, is the ratio of deformation at a given response level to deformation at yield response. This ductility is mainly dependent on the reinforcement and axial load level. High axial loads reduce ductility due to the compressive strains in the concrete, whilst high reinforcement area and yield strength have the opposite effect. Also, transverse reinforcement increases the confinement of the concrete and thus deformation capacity.



Reinforced concrete



Ductility = A displacement at 85% of yield
B displacement at yield

For comparison purposes the Bertero and Mahin approach as recommended by Park (1) can be used as shown in the diagram above. Failure is taken to be 85% of the ultimate load capacity on the descending branch, i.e. it is deemed that at this point the member is no longer capable of supporting design load levels.

Earthquake-resistant structures are now designed following this “capacity design” philosophy, recently implemented in several seismic design regulations worldwide, including Eurocode 8 (2). In this the structure is viewed as comprising two types of zone, dissipative and non-dissipative.

The dissipative zones are designed first and carefully detailed to possess maximum ductility. Then their likely over-strength is estimated. Over-strength arises for a number of reasons, such as higher concrete compressive strength, confinement, and higher yield strength of the reinforcement steel.

The non-dissipative parts of the structure are then designed to withstand the forces consistent with the strength of the dissipative parts, including over-strength.

Analytical studies (3) have confirmed the efficiency of this “capacity design” philosophy, whilst others (4) have used non-linear dynamic analysis to confirm that the Eurocode 8 design codes lead to appropriate failure modes, (plastic hinge restricted to beams and ground floor columns)

There is one important proviso which is reflected in the Eurocode 8 guidelines. This is that the reinforcement steel used should have an actual yield strength which is no more than a certain amount over its nominal yield strength (25% for the “medium” ductility class and 20% for the “high” ductility class products). This is perhaps because the design calculations for the dissipative zones are so dependant on actual ductility of beams, etc.

Alexandrou (5) showed that realistic variations in yield strength of reinforcement, as supplied to site by producers, can lead to enormous effects on the global behaviour of a structure. Other studies (6), (7), (8) have shown that the limits of 20% and 25% over-strength in the steel required by the code are difficult to comply with in practice. Tests on steel from 14 manufacturers (9), (10) showed that no steel was then available which met the requirements for “high” ductility structures, and a large quantity had difficulty meeting the “medium” ductility class requirements.

2.3 Seismic Fuse Inserts

Seismic fuse insets are lengths of high tensile steel of precisely known properties which have been machined to high tolerances. These are fitted into a reinforcement frame using a full performance joint such as a screwed coupling. It is by virtue of their precisely known properties that the seismic design codes can be implemented. Designers need no longer be concerned by the over-strength factors arising from over-strength in the reinforcement. Similarly constructors on site can be spared the burden of testing every batch of steel to ensure that it conforms to the over-strength limit. The logistics nightmare of returning non-compliant batches can also be avoided.

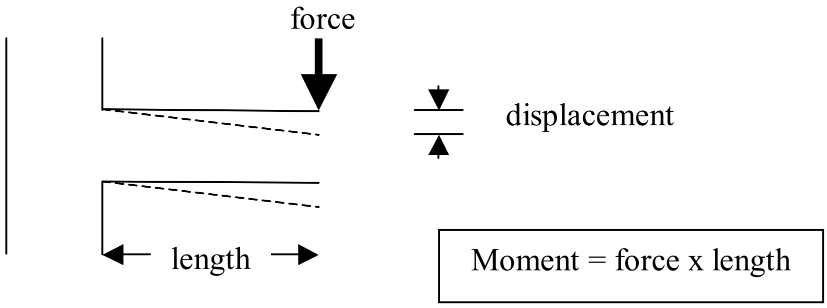
This idea was progressed by Dee Associates who commissioned a series of laboratory trials and a theoretical computer study. These showed positively that such a system has the desired effect in eliminating over-strength in reinforced concrete members. It also showed that the strategic positioning of insets in a structure can create the type of “plastic hinge” required to achieve the levels of ductility prescribed in the seismic design codes.

The invention is now encapsulated in various patent applications (11).

US Patent Application No 09/673,060
European Patent Application No 99914664.0

2.4 Experimental Results

The experimental programme recently carried out was described in more detail in a paper presented to the Conference On Earthquake Resistant Engineering Structure, at Catania, Italy (12). The tests examined the response of various models of reinforced concrete member (e.g. beam) to forces acting at a distance from a fixed end (e.g. column – beam connection).



Three sets of experiments were carried out:

i) no inserts used, continuous rebar only
ii) inserts of similar tensile strength to the rebar (higher than the nominal value)
iii) inserts with 66% of the rebar yield strength.

The output was a series of load versus displacement diagrams such as the one shown in section 2.2. above. As expected, the results for the similar strength inserts showed no failure of the inserts and a load/displacement curve broadly similar to the base case (no inserts),

The results for the models with 13mm inserts, i.e. with a yield strength lower than the rebar were more interesting since these tests had been designed to ensure failure took place in the insert section rather than the rebar. Thus the effects of inserts could be compared with the base case (no inserts). The actual moment capacity was derived from the yield measurement for each case and compared with the nominal values derived from the concrete and reinforcement nominal values. The over-strength factors were then obtained (as the ratio of actual to nominal values of yield strength). As expected, the base case model (no inserts) showed a typical over-strength of 19%, whilst the models with inserts had none. (A comprehensive internal report provides insights into the results of additional tests not made available publicly, concerning the higher strength inserts (13)).

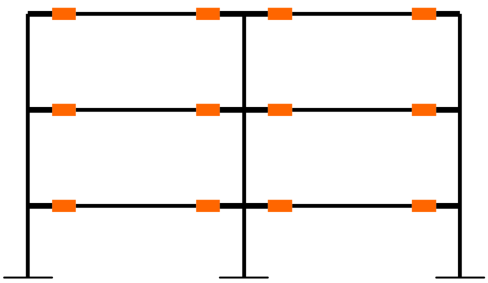
Summary of Experimental Results

	Model No.	**1**	**13 - 2**	**13 – 3a**	**13 – 3b**
	Inserts of 66% yield strength of rebar	**Without inserts**	**2 inserts**	**4 inserts**	**4 inserts**
	Ultimate strength				
a	Horizontal load (kN)	68.5	56.1	55.1	54.6
b	Horizontal displacement (mm)	26.1	19.7	17.7	16.7
	Displacement ductility				
c	Yield displacement (mm)	11.0	10.3	9.4	9.1
d	Failure displacement (mm)	49.0	32.5	33.6	33.5
e	**Ductility = d/c**	**4.5**	**3.2**	**3.6**	**3.6**
	Over-strength				
f	Experimental moment capacity (kNm)	79.6	65.0	63.8	62.8
g	Nominal moment capacity (kNm)	67.5	65.0	63.0	62.0
h	**Over-strength = f/g**	**1.19**	**1.0**	**1.0**	**1.0**

2.5 Theoretical Studies

As discussed earlier, modern seismic design codes address the issue of over-strength by using capacity-design factors to increase the values of beam capacity as initially designed. This ensures that adjacent columns can then be conservatively designed to possess higher capacity than the upper bound value determined for the beams. However, such an approach is not economical since vertical members are in this way over-designed to take into account scenarios that may not occur. In a multi-storey building, such an approach leads to considerable increases in construction costs.

The need for large over-strength factors to account for the variability in material properties can be avoided if a system of controlling the capacity of beams in the vicinity of beam-to-column connections is devised. More specifically, strategic distribution of structural "fuses", as shown below, drastically reduces the uncertainties associated with the flexural capacity of beams in a multi-storey structure. With this procedure, high level of control over the failure mode of the structure is ensured without the need for heavy column over-design.



Structural “fuses” used to achieve the correct capacity-design of reinforced concrete is encapsulated in the Patents referred to earlier, held by Dee Associates. Similar concepts are already accepted for use in steel structures, whereby the greater reliability of behaviour of steel-to-column connectors is assured by reducing the moment capacity of a beam at the joint. This is achieved by “dog-bone” connections first proposed by Plumier (14) and further developed by Chen & Yeh (15). Recent experimental and analytical work carried out by Popov, et. al has further emphasised the advantages of such design philosophy.

As part of the “Seismic Fuse” project, analytical work was carried out using a Finite Element program developed to deal with highly non-linear problems. This model (16) has been used in a very large number of research projects and industrial analyses, and results are routinely verified against test data.

The model incorporates a series of refinements developed to meet certain key requirements:

i) accurate estimation of stiffness and strength under cyclic loading at any strain level

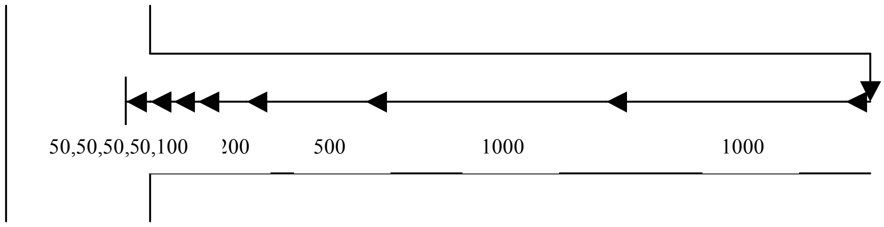
ii) stability and accuracy at very high strain levels, such as the collapse load of the whole structure

iii) account accurately for the effect of confinement

The model by Mander et al (17) was improved to enable the prediction of continuing cyclic degradation of strength and stiffness as well as better numerical stability under large displacements.

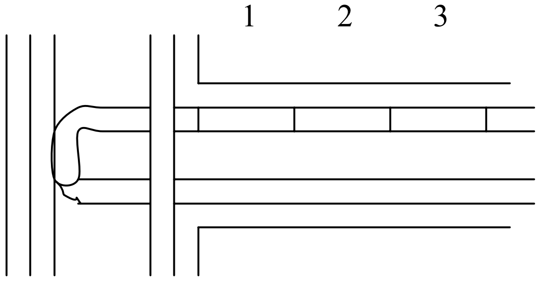
The complex constitutive relationships of early models proposed by Petersson & Popov (18) capture the behaviour of mild steel under variable amplitude. Within the model a number of surfaces enclosing the yield surface are each associated with the increment of plastic strain .The expansion/contraction (isotropic hardening) and translation (kinematic hardening) of these surfaces are governed by pre-specified hardening rules. The isotropic hardening is expressed as a function of accumulated plasticity. A weighting function dependent on the cumulative plastic strain is applied to the virgin and cyclic stress/strain curves to obtain initial and current sizes of the loading surfaces. These curves are represented by five cubic polynomials over five adjacent intervals and a straight line at the end.

A typical office building beam-column assembly was considered, with the cross section at the beam-ends designed according to Eurocode 8. The FE mesh shown below, consisted of nine reinforced concrete cubic elements capable of modelling progressive cracking, and spread of inelasticity as described earlier. Two criteria were used to define element size, viz, to ensure sufficient refinement close to the beam-column joint, and to provide enough flexibility to model several configurations of inserts.



A total of ten push-over analyses were carried out. Three of these were used to study the beam without inserts using the nominal, mean and maximum yield stress of the rebar, the latter two values being taken from the study of rebar variability by Pipa (8) referred to earlier. The remaining analyses were used to study seven configurations of Seismic Fuse inserts, each with yield properties equal to the nominal values for the rebar.

Configuration of Seismic Fuse inserts: Positions 1,2 & 3



Config. No.	**Position 1**	**Position 2**	**Position 3**
1	Insert		
2		Insert	
3			Insert
4	Insert	Insert	
5		Insert	Insert
6	Insert		Insert
7	Insert	Insert	Insert

The results below show that independently of the configuration, inserts are successful in guaranteeing that the capacity of the beam (both at yield and failure) corresponds to what was envisaged in the design.. For these cases, the ratio of the actual to nominal moment at the support is kept to 1.0, whereas the beam without inserts has up to 43% excess yield moment.

Moment at Support – kNm

Configuration	**Yield moment My**	**Ultimate mom Mu**	**My / My nominal**
No inserts - Nominal rebar	205	235	-
- mean rebar yield	237	262	1.16
- max rebar yield	293	317	1.43
Inserts - configuration no:1	205	235	1.00
2	205	239	1.00
3	207	243	1.01
4	205	235	1.00
5	205	239	1.00
6	205	235	1.00
7	213	235	1.04

Although variation in size and location of inserts has little effect on the flexural capacity of beams, the deformation capacity (important in the capacity design of reinforced concrete beams) is affected:

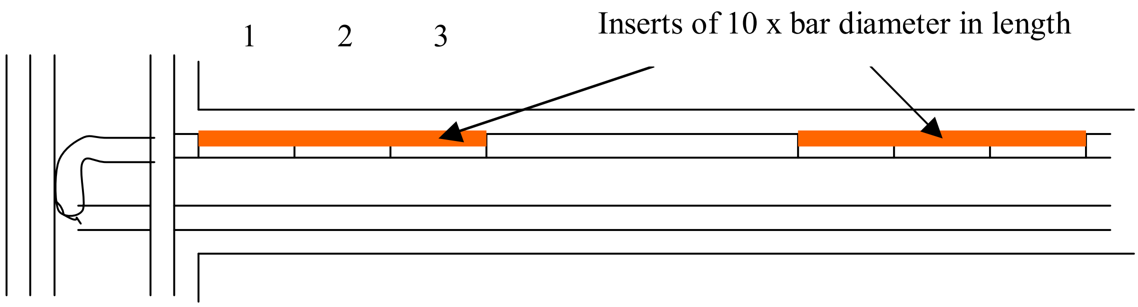
Deformation Parameters

Configuration	**Curvature ductility**	**Plastic hinge length - mm**	**Displacement ductility**
No inserts - Nominal rebar	18.6	380	3.2
- mean rebar yield	13.6	291	2.4
- max rebar yield	9.4	230	1.7
Inserts - configuration no:1	18.6	50	1.9
2	16.5	100	1.8
3	16.2	150	1.8
4	16.2	100	2.3
5	16.2	150	2.3
6	16.2	150	2.2
7	15.2	150	2.5

Thus inserts enable the curvature ductility of the nominal rebar to be more or less maintained, although the plastic hinge is limited to the position of the inserts (i.e. 150mm as compared to 380mm).

The study suggests that inserts should best be positioned with a space between them, or a space distant from the joint. Also, the 380mm plastic hinge of the nominal rebar might be achieved by alternate spacing of large inserts (i.e case 7, three spaces, case 7, etc).

Possible configuration to give plastic hinge equivalent to nominal rebar



3 Advantages of Seismic Fuse Inserts

3.1 Cost Saving

The inserts are not in themselves expensive when compared with some of the other devices on the market. Moreover, they enable low cost indigenous building methods and materials to be used (reinforced concrete).

Seismic fuse inserts can drastically reduce the cost of buildings in seismic zones because “over-strength” is eliminated. Beams will meet seismic design requirements, and columns will be designed accordingly.

If no inserts are used in the beams, they will be stronger than design because the rebar is stronger than its nominal value (by an unknown amount for each beam). The columns will have to be much stronger than the theoretical design to take account of this, hence the considerable unnecessary expense.

3.2 Safety

Seismic fuse inserts are designed to meet the designers specification. Their purpose is to ensure that “over-strength”, which is inherent in reinforced concrete, due to the variability in rebar properties, is eliminated. Therefore there can be no risk to safety as far as the inserts are concerned.

On the other hand, if “over-strength” is not accommodated, and fuses are not used, the structure will be in danger of failing in an undesirable way, possibly catastrophically. This could cause many deaths.

3.3 Simplicity

The idea is simple and all that the designer needs to be concerned about is where they can be positioned to give maximum benefit.

3.4 Compliance

One of the key requirements for seismic design is that there is continuity in the reinforcement such that column rebars are joined to beam rebars by for example couplers. Use of inserts can create a discipline and control during construction to ensure the seismic design codes (Eurocode 8) are adhered to at site level.

3.5 Repair and Renovation

Yield will occur in the inserts since they are weaker than the rebars. Because they are fitted by means of couplers, repair is far easier than when replacing whole members. This may be particularly important when using inserts at the base of columns to absorb energy. After mild earthquakes modest damage may be repairable reducing the need for complete demolition and rebuilding.

References

1 Park, R., Ductility evaluation from laboratory and analytical testing. *Proc of the Ninth World Conference on Earthquake Engineering, Tokyo-Kyoto, Japan, Vol VII, pp.* 605-616, Balkema, Rotterdam, 1998.

2 Eurocode 8: Design provisions for earthquake resistance of structures, *European Prestandard ENV 1998,* European Committee for Standardisation, Brussels, 1996.

3 CEB 1996 . *Seismic Design of Reinforced Concrete Structures For Controlled Inelastic Response – Bulletin 231,* Thomas Telford, London.

4 Coelho E. and Carvalho, E.C. 1991. *Cooperative Research Programme on theSeismic Response of Reinforced Structures – Nonlinear dynamic analysis, Case , Direction y,* National Laboratory for Civil Engineering (LNEC), Lisbon, Portugal.

5 Alexandrou, E. 1991. Seismic response of RC frames with material variability, *M.Sc. Dissertation,* Civil Engineering Department, Imperial College, London, UK.

6 Manzocchi, G.M.E. and Elnashai, A.S. 1994. Characteristics of reinforcing steel in the United Kingdom. *P.R.E.C.8 – Imperial College Report,* Engineering Seismology and Earthquake Engineering Section, Imperial College, London, UK.

7 Plumier, A. and Vangelatou, O. 1995. Synthesis of the statistical data on reinforcing steel collected in Europe and comparison to the Eurocode 8 requirements. *PREC8 – Liege Report No 4,* Liege Civil Engineering Institute, Liege, Belgium.

8 Pipa, M. and Carvalho, E.C. 1995. Reinforcement steel characteristics for earthquake resistant structures. *Proc. of theTenth European Conferenceon Earthquake Engineering, Vienna, Austria.* Edited by G. Duma, pp. 2887-2892, Balkema, Rotterdam.

9 Pipa, M. 1993. Ductility of reinforced concrete members subject to cyclic loading – Influence of reinforcement mechanical characteristics. *Ph.D. Thesis,* National Laboratory for Civil Engineering (LNEC), Lisbon, Portugal.

10 Adriano, T. and Park, R. 1986. Seismic design considerations of the properties of New Zealand manufactured steel reinforcement bars. *Bulletin of the New Zealand National Society for Earthquake Engineering* **19**(3): 213-246.

11 International Patent Application No PCT/GB99/01040
US Patent Application No 09/673,060
European Patent Application No 99914664.0

12 Goodfellow, R., Pinho, R., Salama, A., and Hancock, M. (1999) "Applications of fuse inserts in seismic design of reinforced concrete structures." *Proc. Conf. On Earthquake Resistant Engineering Structure,* ERES 99, Catania, Italy.

13 Development of Fuse Inserts for Ductile Seismic Behaviour – Experimental Description and Results – Report for Dee Associates 1999.

14 Plumier, A , "Modelling and design of steel structures in the Eurocode 8 context", *Experimental and Numerical Methods in Earthquake Engineering*, 319-349, ECSC, EEC, EAEC, Brussels and Luxembourg, 1991.

15 Chen, S.J.& Yeh, C. H. "Enhancement of ductility of steel beam-to-column connections for seismic resistance". *Proc. Of the SSRC 50th Anniversary Conference*. Lehigh University, Bethlehem, Pennsylvania, 1994.

16 Martinez-Rueda, J.E. & Elnashai, A.S. "Confined concrete model under cyclic load", *Materials and Structures*, Vol 30, No 197, 139-147, 1997

17 Mander, J.B., Priestly, M.J.N. & Park, R., "Theoretical stress-strain model for confined concrete", *Journal of Structural Engineering*, ASCE, Vol 114, No 8, 1804-1826, 1988.

18 Petersson, H. & Popov, E.P., "Constitutive relations for generalised loading", *Journal of the Engineering Mechanics Division*, ASCE, Vol 103, No 4, 611-627